VOLUME 81 PAPER No. 830

PROCEEDINGS

AMERICAN SOCIETY
OF
CIVIL ENGINEERS

NOVEMBER, 1955

LOAD TEST OF A DIAGONALLY SHEATHED TIMBER BUILDING

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STRUCTURAL DIVISION

{Discussion open until March 1, 1956}

Headquarters of the Society
33 W. 39th St.
New York 18, N. Y.

PRICE $0.50 PER COPY

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This paper was published at 1745 S. State Street, Ann Arbor, Mich., by the American Society of Civil Engineers. Editorial and General Offices are at 33 West Thirty-ninth Street, New York 18, N. Y.

LOAD TEST OF A DIAGONALLY SHEATHED TIMBER BUILDING

J. Morley English,[1] A.M. ASCE
and C. Franklin Knowlton, Jr.[2]

SYNOPSIS

A description of the testing of a diagonally sheathed timber building is presented. The building, 200' long by 50' wide, and one story high is located in Log Angeles County, California. It was designed in accordance with the county building code—except that the end wall shears were in excess of allowables for single diagonal sheathing and the length-depth ratio of the diagonally sheathed roof was in excess of that normally allowed. The building was accepted for occupancy by the County Building Authorities on the basis of the proof tests described herein.

The structure was extensively instrumented to record as many deflections and local strains as practicable. Loads were applied horizontally at each of nine panel points up to values of 1-1/2 times design wind loading. The test results are recorded in graphical form and the load deflection characteristics described. The theoretical action of diaphragms is discussed.

INTRODUCTION

Tests were conducted on a timber diaphragm building to determine its loading and deflection characteristics. The building is located at 1643 West 135th Street, Los Angeles County, California, and was tested during construction on August 30, 1952.

Countless efforts have been made to obtain information pertaining to the proper methods of determining the strengths and stiffnesses of wood sheathing laid at an angle of approximately 45° to the boundary members. Attempts have been made also to establish a fixed strength of a combination of these materials when functioning as a structural unit by considering the entire assembly as a structure of predetermined strength—regardless of the internal construction. For example, some sources have recommended that the allowable structural strength of a diagonally sheathed diaphragm be 300#/linear ft.

In certain respects this type of solution would be similar to one which the designer attempts to establish a fixed strength for a steel bridge of given shape by specifying a predetermined value of strength of a structure of this shape without regard to the design of the elements. It is just as logical to approach the design of the bridge in this manner as to establish a fixed strength of a diagonally sheathed diaphragm.

Since all of the tests on single-layer diagonally sheathed diaphragms that had come to the attention of the authors had been made to establish gross strength, it was decided to try to obtain as much information as possible on a

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full sized building diaphragm. An opportunity arose to test a building in which the length and design loads were both in excess of those accepted in general practice.

The building was designed on the same principle as that of a full tension field web beam, i.e., Wagner beam. In accordance with this principle the following assumptions were made:

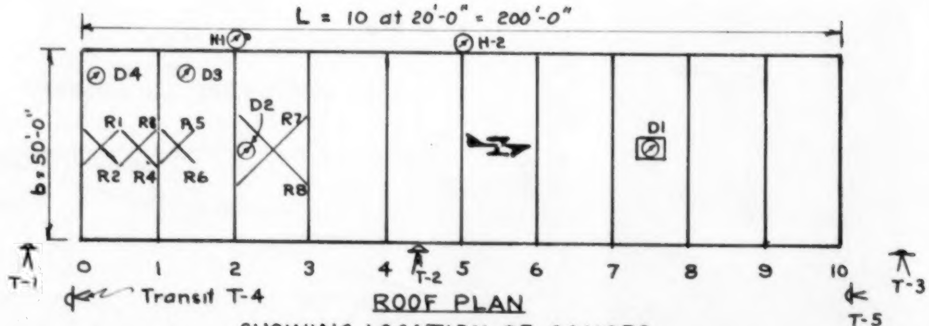
1) The distribution of loads is uniform in all sheathing boards;
2) All forces are transmitted parallel to the interior elements;
3) The boundary members must resist the components adequately to develop the axial loads of the sheathing boards (both axial and normal to the axis of the boundary members).

Particular attention was paid to the design of all connections.

Description of the Building

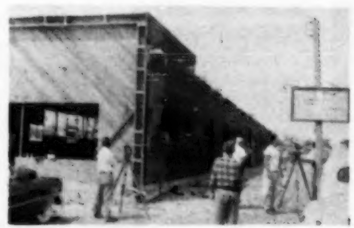
The building was of wood frame construction, 200' long, 50' wide, with wood curved chord trusses spanning 50' across the building. Fig. 1, Fig. 2. They were installed at 20 foot center-to-center spacing and thus provided ten 20' x 50' bays. The trusses rested on wood posts, which in turn rested on concrete foundations. The roof sheathing was laid at 45° to the joists with the joists spanning between the trusses. The joists were set in hangars. The long walls were designed to be of a minimum type construction, 2 x 4 vertical studs spaced at 16" centers. The studs rested on a wood sill bolted to a continuous concrete foundation. At the top they were connected to a wall plate which was continuous between trusses. By means of metal connections, the wall plate was bolted to the trusses and to the posts which supported the trusses. A splice between the wall plates was provided by extending the truss bearing plate. The building paper, exterior stucco, and interior finish of the building had not been applied at the time of the tests. A horizontal girt member was installed at the sheathing line and flush with the top of the trusses. This girt member extended continuously between the trusses at the heel and was designed to provide strength and stiffness at the boundaries of the roof diaphragm. The design loads used to determine the sizes of the girt members were obtained by assuming that the shear load in any panel was distributed uniformly between the sheathing boards from wall to wall. This, then, introduced an axial load and a bending load into the girt member. An exceedingly large girt member would have been required in the end two bays had not an intercostal strut of blocking been placed at the midspan of the joists. The intercostal was tied by a steel strap extending from wall to wall to ensure that both tension and compression forces could be carried. Nailing was specified to withstand the loads introduced into the girt from the sheathing. The lateral loads were all taken by the end walls. Since a number of openings for windows and doors was required, the end wall shear intensities were high, more than double that for the end roof panels. The window and door opening were reinforced to provide for the high stresses.

In the back wall, Fig. 3, the piers (b) were all of uniform size. This made the problem relatively simple since all could be assumed to carry the same shear intensity. It was also assumed that the total shear was constant across the building. Consequently, the shears in sub-panels d and f were determined by taking a vertical section through a window, and in sub-panels b by a horizontal section through the windows. Now by elimination the shears in sub-panels c and d were established. The change in shear intensity from one



ROOF PLAN
SHOWING LOCATION OF GAUGES
(a)

R-7 SCALE
(b)



TRANSIT GAUGE SET-UP
(c)

D-2 SPLICE GAUGE
(d)

FIG 1

INTERIOR VIEW OF TEST SET UP
(a)

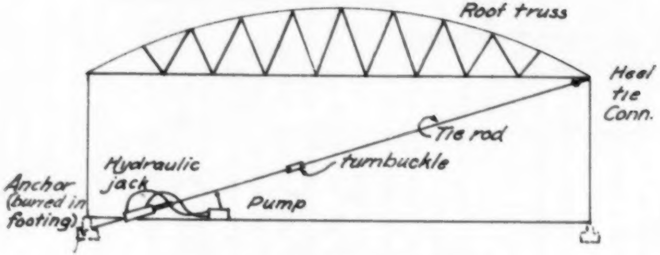
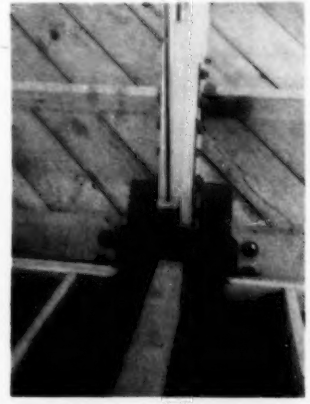


DIAGRAM OF TEST LOADING SET UP
(b)



HEEL TIE CONNECTION
(Looking up)
(c)

PUMP & JACK SET UP
(d)

FIG 2

sub-panel to another was provided for by the bending resistance of the posts and sills which bounded the windows and which extended through the sub-panels. Continuity of axial force through the intersections was provided by metal gusset plates and straps.

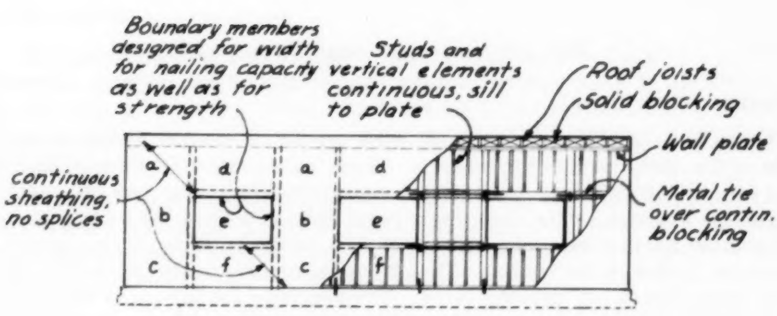
The same basic assumptions were made for the design of the front (south) wall of the building, Fig. 4. However, the piers were not uniform so that it was necessary to make an assumption concerning their stiffness. In the absence of better knowledge these stiffnesses were computed on the basis of the shear and bending deflections of a homogeneous material. In the light of subsequent tests such practice is open to question.

It was discovered during construction that the builder had made a faulty installation of the tie between the roof and front wall diaphragms. This necessitated placing a sheet metal tie between the structural elements of the roof and wall. The modified connection proved satisfactory.

The materials employed were no better than used in conventional construction. However, some care was taken to see that the nailing was done according to specification, particularly at points of high load intensity.

NORTH END OF BUILDING

(a)



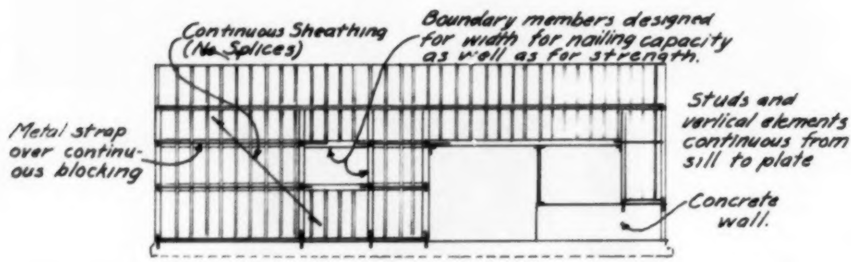
STRUCTURAL ELEMENTS - NORTH END

(b)

FIG. 3

SOUTH END OF BUILDING

(a)



STRUCTURAL ELEMENTS - SOUTH END

(b)

FIG 4

Test Equipment and Instrumentation

1. Loading (See Fig. 2)

Loads were applied at each panel point through a bracket welded to the shoe at the heels of the roof trusses. These connections were located on the east side of the building. Anchorages were provided at the time of construction in the footings on the opposite or west side. Between the footing anchorage and the heel of the truss, diagonally across the building, a tie rod and hydraulic jack was introduced. Thus by applying load to the jacks the rods were tensioned to any desired value. The pressure for the jacks was provided by a series of hand operated pumps. Since only six pumps were available the jacks were manifolded in pairs for panels 1 and 3, 4 and 6, 7 and 9. Individual jacks were provided for panels 2, 5 and 8. This was done in order to run three tests under different loading arrangements as follows:

Test No. 1 - Two point loading at panels 2 and 8.
Test No. 2 - Single point loading at midspan.
Test No. 3 - Distributed loading at all panel points.

During test #2 the center bracket fractured at the truss heel due to a faulty weld. It was repaired, but not before test three had progressed to the 4,000 lbs jack load. As a consequence test three represented an 8 panel loading except for the last two load increments when the 9 panels were fully loaded.

Loads on the jacks were indicated by the calibrated pressure gauges on the pumps. The maximum load of 6500# on all jacks was limited by uplift on the footings at the wall.

2. Instrumentation

Measurements of the deformation in the structure were made in four ways.

a) Transits were set up at the Northeast and Southeast corners, one pair to sight along the NS building line on six inch rules which were fastened to the side of the building at the truss heels, and also at intermediate points in the end panels, Fig. 1. A third transit was placed along this line to read the points near the center. A second pair was sighted West to record the N.S. deflections of the corners.

b) Dial gauges were located as follows:

1) On the end shear walls to record deformations perpendicular and parallel to the sheathing over gauge lengths ranging from 40" to 60", Fig. 5. This was attempted by nailing an indicator board on the wall in such a way that movement at the far end was recorded on the gauge as the accumulated deflection over the length of the board. Since only one end of the board was nailed to a sheathing board it was not possible for the indicator board to be stressed. However, rotation in the sheathing could occur and thus render it impossible to determine the component of deflection due purely to translation by the reading of dial A.

It had been planned to locate indicators running parallel to the sheathing as well as those perpendicular but there were a limited number of gauges available. Unfortunately the arrangement used was not as successful as others that could have been used. But this view is in retrospect.

2) Dial gauges were located over splices in the roof sheathing boards, Fig. 1d, and similarly over 2 bolted splices in the chord members at the truss heel so instrumented.

3) A dial gauge was located in a roof vent opening on the crown midway between panels 7 and 8, Fig. 6b, and reading the deflection of the crown from the diagonal sway frame. This did not prove particularly successful since the deflection of the roof under the weight of the observer occasioned some variability of reading.

4) A dial gauge was located at the sill mid way between the corners of the North shear wall. It was fastened a distance out from a stud to read deflection from the sill, and measured the tendency for the stud to lift up due to compressive load in the sheathing.

c) Large gauge length deflections were taken by direct scale reading on diagonal wires. These wires were fastened rigidly at one end and through a light pretensioned coil spring at the other. Thus deformation in the structure which occurred over the length of the wire would all be reflected in the movement at the spring (assuming the wire infinitely stiff by comparison to the coil spring). These gauges were located diagonally across the ends of the building and also in three 90 degree crosses placed at 45 degrees with the joists in panels 0-1, 1-2, 2-3 Fig. 1a and 1b.

d) Electrical strain gauges (SR-4) were located on the steel straps in the south wall at the blocking which ran above and below the windows between the door and Southwest corner of the building. These gauges were located at each end of the strap. Fig. 6a.

3. Tests

Test 1. - Panels 2 and 8 loaded.

This test proceeded normally. No undue distress was noted anywhere in the building although slight noise indicated some slip was occuring in nailed joints.

Test 2. - Proceeded normally to the 4,000 lb loading. As additional load was attempted the temporary bracket for the tie rod connection failed due to a faulty weld. The test was discontinued.

Test 3. - For the first portion of this test, up to and including the 4,000 lb value per jack, panel number 5 was inoperative so that only 8 jacks were loaded. For the final loading number 5 was again functioning. The building was loaded to 6,000 lbs per jack, the maximum value which had been anticipated. Since no sign of any distress was evident it was decided to add an additional increment of 500 lbs per jack. Although noise was evident throughout the tests and also perceptible creep occurred under the higher loadings, no sign of failure was evident anywhere in the timber elements. However the steel straps over the blocking at the window sill height adjacent to the door suddenly buckled when the load reached the 5000 lb value. This was a compression region which indicated that the straps were carrying the load rather than the blocks which had been designed for that purpose.

The results of the three tests are shown in the charts of figures 7 through 12.

The loading curves are all plotted on an ordinate of panel point loading. Since only 8 panels were loaded during the first loading increments, this method of plotting tends to distort the picture. If the ordinates had been chosen as end shears the values for the 6,000 lb and 6,500 lb panel loads would be 10% higher on a relative scale.

Discussion of Test Results

The tests revealed that the building was adequately strong to withstand the design loads without excessive deflection and no serious permanent set. The loads imposed were one and one-half times design wind load and represented a maximum average shear of 585 lbs per foot in the end walls, without allowance for doors and windows. The concentration which occured due to the large open spaces meant that the average in the shear resisting elements must have been about 1400 lbs per foot.

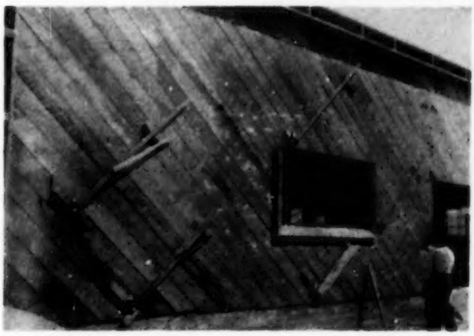
Apart from revealing the capacity of the building to withstand the loading—which was expected as a matter of course—the tests supplied a great deal of information on the deflection of diaphragms and deformation of the component parts. In general the loading of tests 1 and 2 were too low to reveal an appreciable deflection. In most cases therefore, the results of test 3 only are plotted. The loading cycle for test 3 was zero to 2000 to 4000 lbs to zero to 5000 lbs to zero to 6000 lbs to 6500 lbs to zero. The last two loads were on all 9 panel points.

Residual deformation remained after each unloading. This is revealed in the plotted curves except for Fig. 7 which shows the horizontal deflection curves for the building measured from the residual deflection rather than the

initial zero. The analysis of the load deflection characteristics have all been based on the load deflection relationship of the last loading cycle from zero to 6000 lbs to 6500 lbs to zero.

The discrepancies in Fig. 7 at panel points 3 and 7-1/2 are due to the fact that three transits were used to take these readings. It may be noted that the center transit readings were invariably low at one end of the range and high at the other. This was apparently due to the creep which occurred in the time interval between the readings of the two instruments on the same point.

In test #1 the region between 2 and 8 was subjected to pure bending. No shear could occur in this region. As a consequence the deflection over the region can only be accounted for by bending. It may be observed that the deflection curve between points 3 and 7 is a straight line. The slope could be explained by the effect of creep during the interval taken to read the transit.

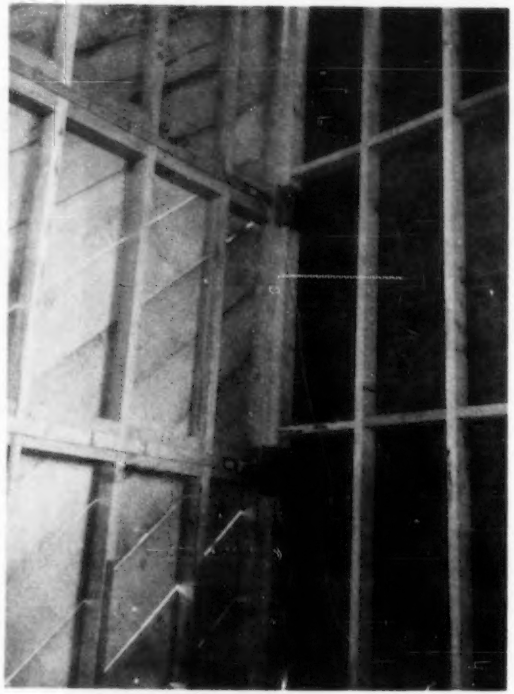


SOUTH WALL GAUGE SET-UP
(a)

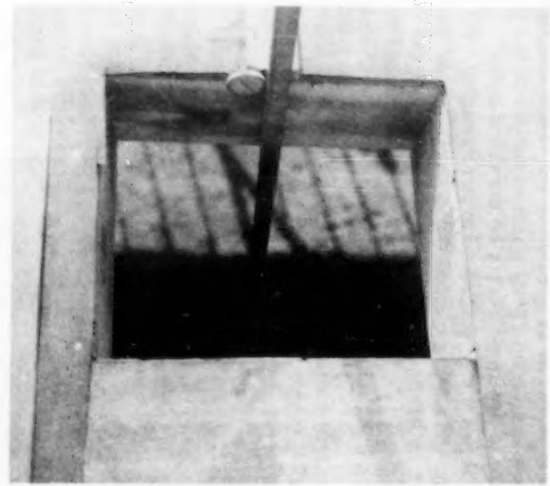
TYPICAL GAUGE DETAIL
(b)

NORTH WALL GAUGES
(c)

FIG 5

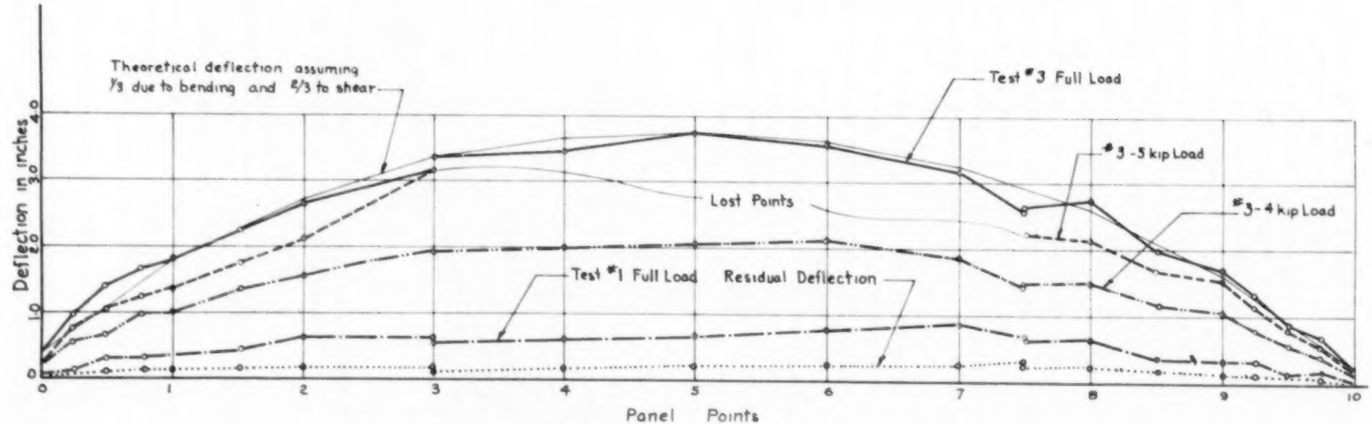


SR-4 GAUGES
S-W Corner
(a)



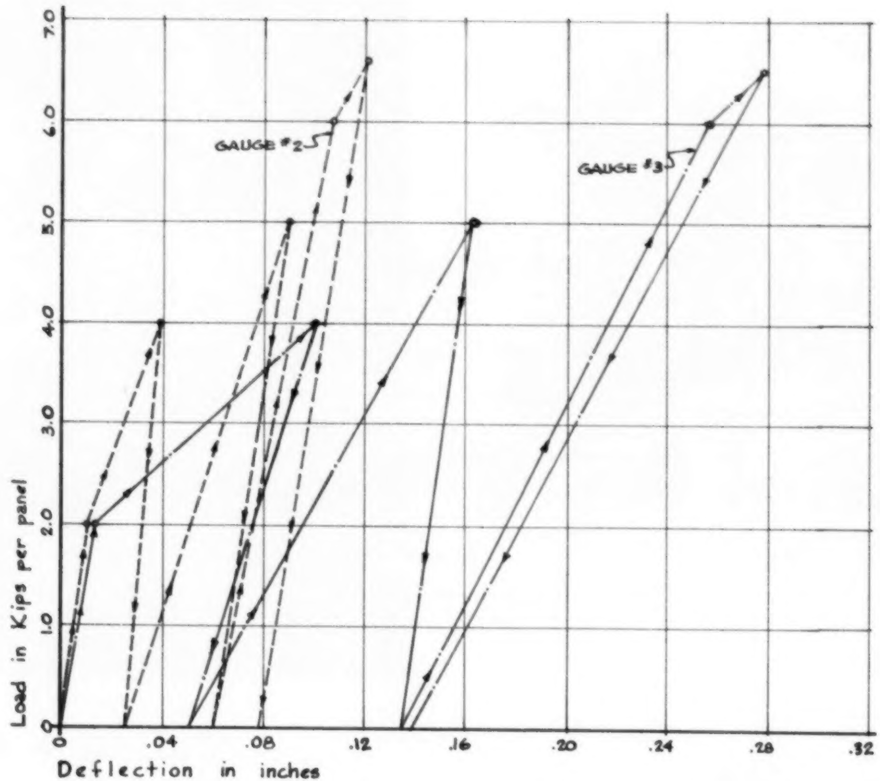
ROOF VENT DIAL GAUGE
(b)

FIG 6



BUILDING DEFLECTION CURVE

FIG 7



LOAD · DEFLECTION *for* HEEL GAUGES

FIG. 8

The irregularity at point 7-1/2 may be explained by bending of the chord between panel points 7 and 8. It may be seen that similar deformations occur in the other end panels but at the South end they are in the opposite direction. The direction of loading was such that the sheathing boards were in tension at the South end and compression at the North end of the building. Up to the 4000 lbs load in test 3 when the center jack was inoperative a region of zero shear occurred between 4 and 6. Again no measurable curvature can be observed in that region. The inference to be gained from these results is that no measurable bending deflection occurred and therefore the observed deflection must have been due to shear alone. This conclusion is partially borne out in later discussion by the correlation of joint displacements and deflection.

Further evidence that the deflection was one of shear rather than bending was obtained from the transits T-4 and T-5. No data is plotted for these since no measurable horizontal corner deflections in the North-South direction could be observed. Bending deflection would have been evidenced by an overall change in length of the chords. For shear deflection the chord lengths would remain constant. The amount of chord shortening observed by the dial gauges at panels 2 and 5 was small and the accumulated amount for the whole building length could have been too small to be recordable by transit.

While the evidence seems to strongly support the argument that the deflection was due to shear alone it must nevertheless be remembered that the determination of the moments or shears from an examination of deflection is a process of differentiation and as such is subject to considerable error.

The horizontal deflection of the building must be accounted for by the deformations and strains which occur within the individual structural elements. That it cannot be accounted for by the elasticity of the lumber is apparent if one considers the deflection of a beam of homogeneous material with a modulus of elasticity comparable to douglas fir—for purpose of illustration say $E = 1.5 \times 10^6$ psi. Assume the effective thickness of the web as 1" and the flange or chord as 100 sq. in. These, while not strictly accurate, would be in the correct order of magnitude. Thus the deflection under maximum loading in test 3 would be less than 0.02 inches. The gross deflection could not be measured so accurately. Consequently the observed deflection must be explained by other deformations than that of the elasticity of the lumber which therefore can be completely neglected as a factor.

Source of deflection

If the elongation of the wood itself does not contribute an appreciable deflection the answer must be looked for elsewhere. There are at least four other ways to account for the deflection.

1) Connections at either end of the longitudinal members—chords and stringers.
2) Splices and end connections of the sheathing due to stress components acting in the direction of sheathing.
3) The squeezing together of the boards by slip in the intermediate nailing and bending of the stringers and sheathing boards.
4) Bending and realignment of boundary members.

These four effects will be discussed separately and in the order indicated.

1) From elementary beam bending theory the deflection of the beam due to an elemental rotational deformation is obtained from

$$\frac{d\theta}{dx} = \frac{M}{K_1} \tag{1}$$

where K_1 is the rotational spring constant corresponding to E1 in conventional beam theory. Fig. 8 shows the deformation for the splice at the truss heels of two joints, panel points 2 and 5. The value of K_1 may be computed using the joint deformation from the angle relation. Taking the moments corresponding to 6500 lb. panel load of test #3, and assuming the joint movements, δ , on the East and West sides as equal, then K_1 as computed for joint 2 is 1.4×10^{10} pounds ft^2, and for joint 5 is 7×10^{10} pounds ft^2. The reason for the one value being 5 times the other may be explained by the distribution into the chord members of the forces arising from the bending of the diaphragm. Near the midspan the chord or boundary may tend to carry a greater percentage of the moment than it does towards the ends of the building.

By assuming the K_1 value for the whole building is that which was determined for panel point 5, the midpoint deflection of the building will be approximately

$$\Delta = \frac{5}{384}\left(\frac{P}{a}\right)\frac{L^4}{K_1} \tag{2}$$

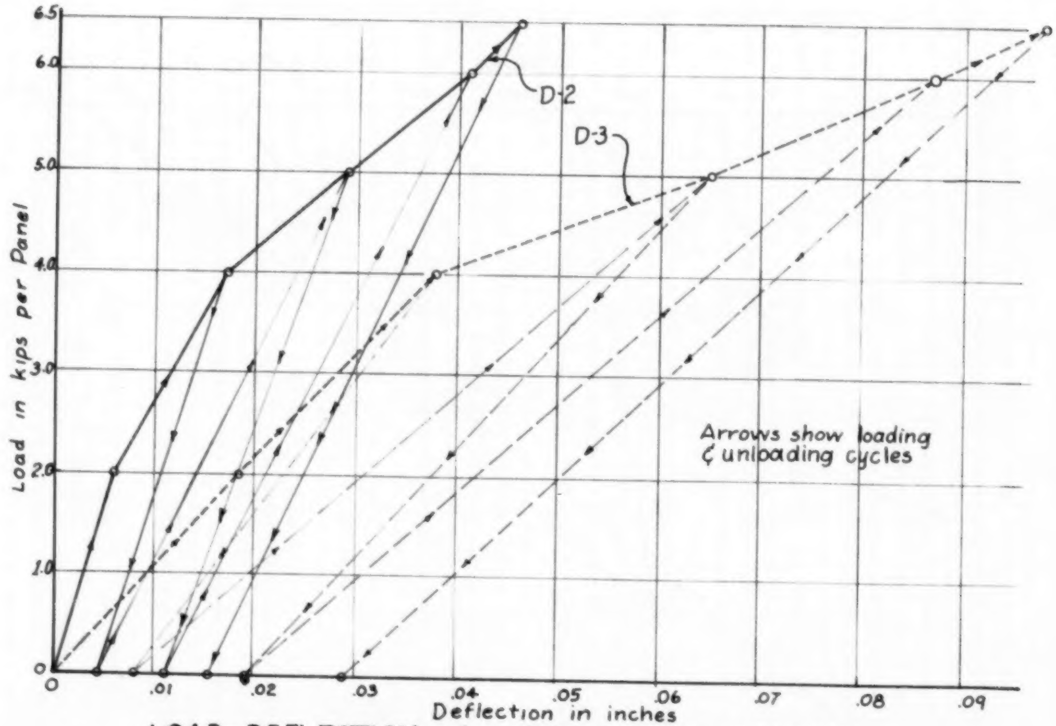
due to bending only. This amounts to 1.16" for the maximum load condition. This then accounts for less than half of the total deflection even when the extreme value of K_1 is used.

2. The deformation in the direction of the sheathing can only occur at the ends of each board. This will depend entirely upon the load-deflection function of the nailed connection. The deflection of the whole structure due to this deformation component will be directly proportional to the number of splices and boundary connections in the sheathing—a most important conclusion since for a given intensity of loading per board the deflection is independent of the width of the diaphragm. Doubling the number of connections for a given width will double the deflection. This means that insofar as this component of deflection is concerned, the length-width parameter is meaningless. A diaphragm with a length-width ratio of 4 may be stiffer than one with a ratio of 1. This appears to be borne out by the Oregon Forest Products report T-6, Oct. 1953.

The elongation of the splice is indicated in Fig. 9 and 10. The rather odd behavior of gauge D-4 was probably due to a realignment of forces after an initial elastic straining had occurred in that region in which the gauge was located. Irregular distribution of loads could very well occur in the corners of the building. In fact, when it is considered that the diagonals in the region of D-4 run across the corner and not into it and also have one end terminating at the chord member, the action is understandable. As long as friction between the chord and sheathing boards is being mobilized the splice at D-4 will be subjected to loads corresponding to those in other regions of the panel. Once some slip of the chord occurs the effect will be an unloading of D-4.

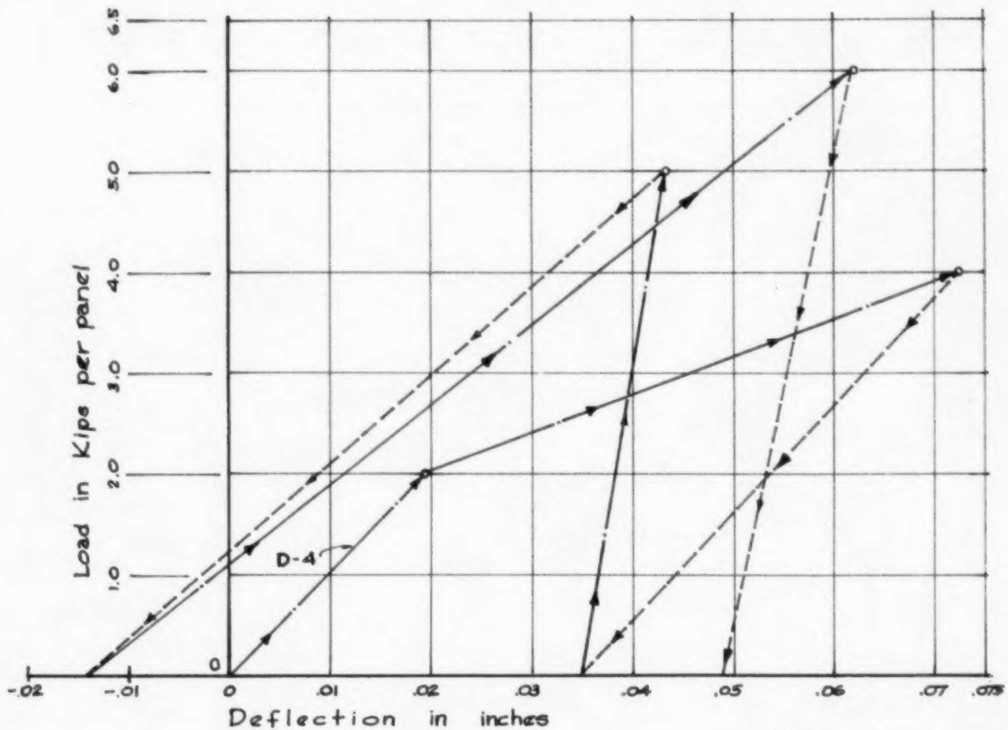
Since the sheathing boards were 18' long there was on the average 1.5 splices per panel along each line of boards. As a consequence the deflection per panel due to the splice is

$$\Delta = \frac{3V}{2K_2} \cos 45^\circ \tag{3}$$



LOAD-DEFLECTION for ROOF DIAL GAUGES

FIG 9



LOAD · DEFLECTION for ROOF DIAL GAUGES FIG 10

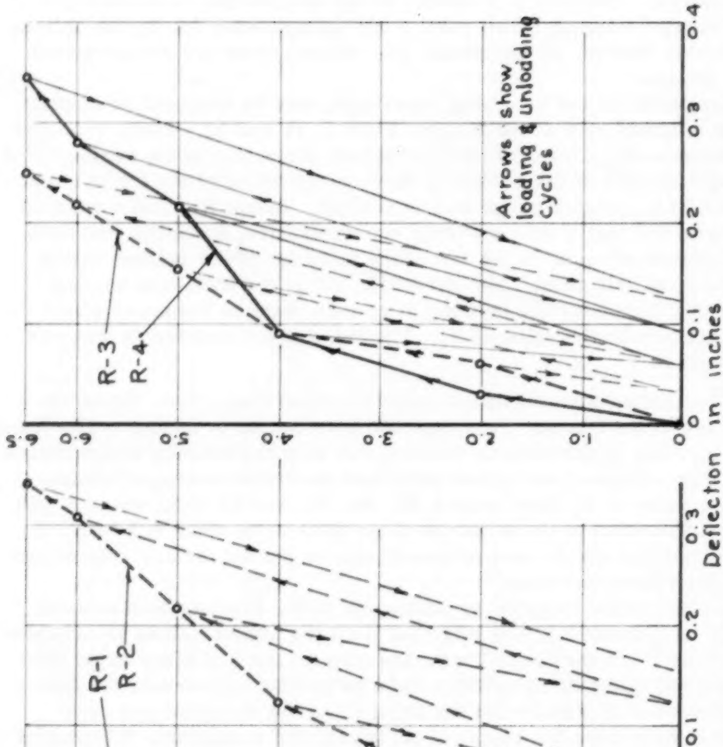
By using the value of K_2 computed from an average of D-2 and D-3, Fig. 9, and assuming that the same average value holds for all splices, the total diaphragm deflection from this source amounts to less than 1/4 inch. The two joints gave quite different values so that it might appear unreasonable to average them. However, one gauge was located at the crown and the other close to the edge. The distribution of shear over the section is probably not uniform and the variation in the properties of each splice sufficiently large to account for the wide differences. The average may more closely represent the effect throughout the whole width. Considering that 80% of the boards in one panel end at the boundary and that each end constitutes one half a splice there will be effectively an increase of 40% for the boundary splices. Therefore, it seems inconceivable that more than 3/8 of an inch of the deflection at panel point 5 can be accounted for by the splices in the sheathing boards. In any event, this follows from the measurement of the dial gauges.

A further check on the foregoing conclusion may be obtained by examination of the diagonal roof scale gauges, Figs. 1, 11 and 12. These readings in effect measure the averaged effect of splice deflection in the region. For those in the direction of the sheathing the average value of the k's is about 135^K/in per 20 ft. panel if gauge R-1 is omitted. Gauge R-1 had a value of 225 and it was felt that it was not truly representative, since the measurement was located so close to the end of the building; fewer splices would occur in the first half panel. The deflection computed from this will not include the 3/2 factor for the number of splices because the gross effect is taken into account by the value of K_2. The deflection computed in this way amounted to 0.42".

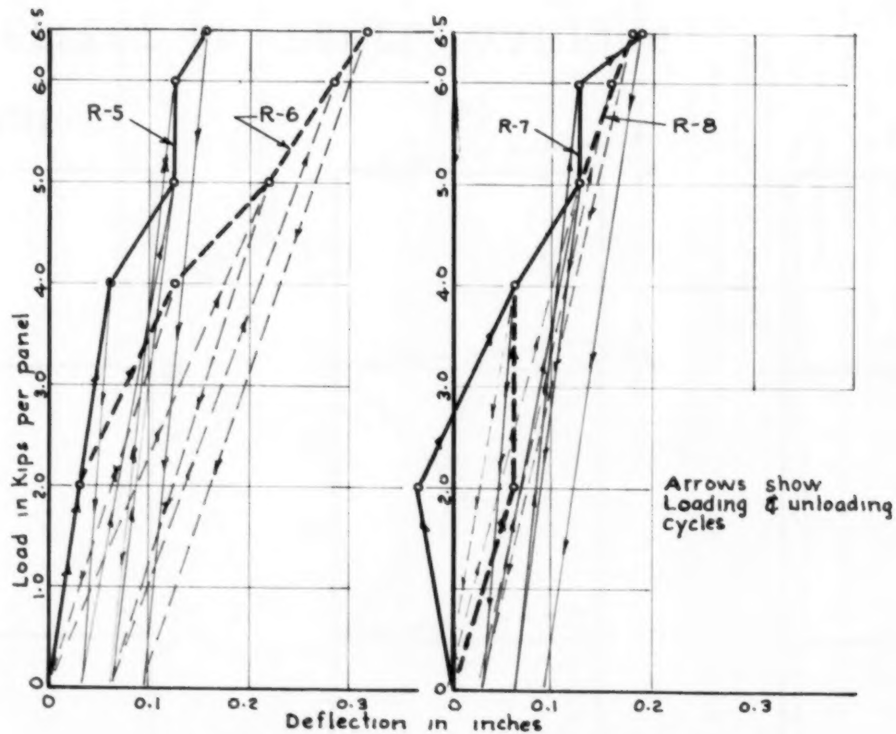
3. and 4. The deflection still unaccounted for must come from the strain perpendicular to the boards. No gauge had been set up to measure this from board to board nor to distinguish between nail slip and bending of the boards and joists. The diagonal wires accomplished this on an averaged basis. The average value of K_2 from gauges R2, R4, R6, and R8 is 92 which would indicate the deflection at the midpoint of the span to be about 1/2 inch. It should be noted that the strains perpendicular to the boards are only slightly greater than those parallel.

The total deflection obtained by adding the three effects would account for about 2" of deflection at the midpoint if all the higher values are chosen and as little as 1", if the lower limits are added. Since slightly more than 3" occurred, the question naturally arises as to what occasioned the difference. Obviously if this extra did not arise from the deformations measured it must have been the result of something not measured. Warping of the section might explain it.

Consider a panel, Fig. 13, with diagonals ab and cd corresponding to gauges on the roof diaphragm. If the frame is deflected the change of length of the diagonals will provide a measure of δ. If no warping can occur as in Fig. 13b the deflection δ_1 measured at the corner of the frame will be equal to δ_2. Thus the diagonal gauge provides a measure of the total deflection. If on the other hand warping can occur as in (c) then δ_1^1 will not be equal to δ_2^1. For the case as drawn the true deflection δ_2^1 will be greater than indicated by the gauges. It is easy to visualize the deflection due to strain in the sheathing splice. It is not so easy to visualize the mechanism of the strain normal to the boards and from this to realize the forces which must be acting. The deflection must all be accounted for in the movement at the ends of the boards and joists since the shortening of a

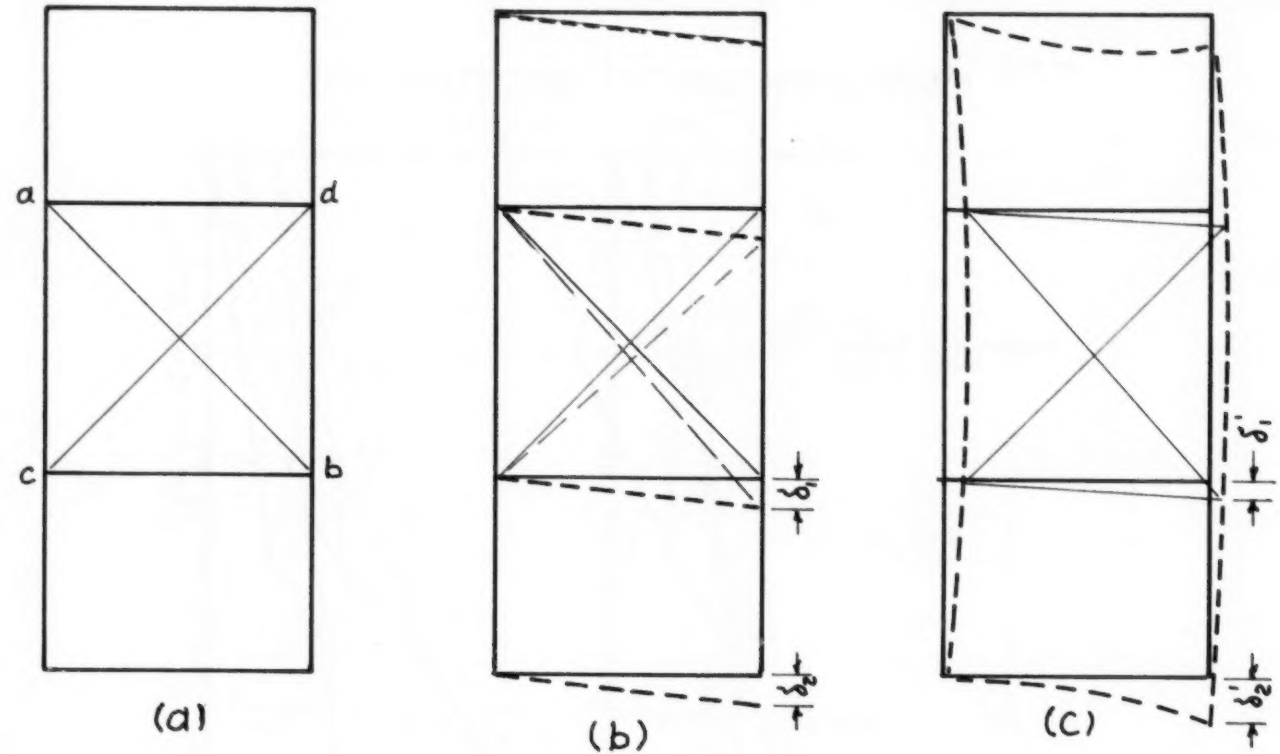


LOAD-DEFLECTION for ROOF SCALE GAUGES FIG 11



LOAD-DEFLECTION for ROOF SCALE GAUGES

FIG 12



EFFECT of PANEL WARPING

FIG. 13

board due to bending can be ignored. The shortening of a joist splice is accompanied by a lengthening of a board splice. Therefore it is apparent that the movement at the ends of the joist must be considerable and will be accompanied by high loads on the nails in that region.

However, the resistance to this movement can be developed in many ways. An examination of the diagram of figure 13 shows that the deflection must also be accompanied by a translation of one diagonal over the other. Similarly, if the boards were not nailed to the joists, the panel deflection would occasion movement transversely. The intermediate nailing serves to inhibit such movement. However, this in turn creates a tendency to bend the boards. Thus is derived some of the resistance normal to the sheathing. Also the rotational effect of the bending will be resisted by the nail couples. It is suspected that the resistance provided by the latter action may be inconsequential.

Finally the bending stiffness provided in the chord members will cause the structure to behave as one with an incomplete unidirectional stress field web. Some shear will be carried in this fashion. If it did not act so, the sheathing boards terminating along the chord would be totally ineffective. Gauge D-4 would not have recorded any deformation.

Comparison of Bending and Shear Deflection

The deflection may be considered as that due to bending and that due to shear. For a homogeneous material the ratio of bending and shear deflection for a uniformly loaded beam is readily computed as

$$\frac{\Delta_B}{\Delta_V} = \frac{5}{96}\frac{GAL^2}{EI} \tag{4}$$

But GA corresponds to the summed effects of 2 orthogonal strains. Thus

$$\frac{1}{GA} = \frac{1}{\sqrt{2}}\left(\frac{1}{K_2} + \frac{1}{K_3}\right) \tag{5}$$

And EI corresponds to K_1

Therefore

$$\frac{\Delta_B}{\Delta_V} = \frac{5\sqrt{2}\ K_2 K_3}{96\ K_1(K_2+K_3)} \tag{6}$$

In a homogeneous material $K_2 = K_3$ and if Poisson's ratio is zero

$$E = 2G$$

and

$$\frac{GA}{EI} = \frac{A_{web}}{2I}$$

For a flanged beam

$$I \approx A_{flange}\frac{b^2}{2}$$

thus

$$\frac{A_{web}}{A_{flange}} = \frac{\sqrt{2}\, K_2\, b^2}{2\, K_1}$$

Substituting the values of the K's determined from tests

$$Aw/Ar = 0.33$$

For a rectangular section

$$I = \frac{A_w\, b^2}{12}$$

Thus $K_1/K_2 = 147$ for our building

The actual ratio on the basis of the maximum for K_2 and the minimum for K_1 is on the order 1000.

The indication from these assumed extreme conditions are that the roof tends to act as a flanged beam. If it were homogeneous the ratio of the K's would have to be almost 10 times the value indicated in the test.

A further emphasis of the point may be made by comparison of the theoretical ratio for shear and bending deflection of a homogeneous beam to that actually observed.

By substituting I for a rectangular section into equation 3 and the L/b ratio for the building equal to 4, Δ_B should be 5 times Δ_V. In the actual case the shear deflection is greater than the bending deflection by an appreciable, if unaccountable, amount.

Since the bending deflection appears not to exceed 1/3 of the total as an outside limit, a theoretical deflection curve based on an allowance of 1/3 due to bending and 2/3 to shear was plotted relative to the actual maximum deflection curve of Fig. 7. It appears to fit very well. A 100% shear deflection curve would have fallen slightly below the measured curve.

End Wall

Data gathered from the gauges on the North and South walls are not presented. The readings were very irregular so that no consistent load deflection relationship could be determined. The only conclusion which might be drawn from this fact is that the distribution of stress in the diaphragm tends also to be very irregular. The stiff points tend to concentrate the resistance until a sudden slip develops to permit unloading to some other point or points which in turn build up to capacity before slipping, and so on, until in the limit or ultimate load condition, all the redundant means by which the structure can support the load are fully mobilized.

One bit of data gained from the dial gauge located to measure strain along a board between the windows of the North wall was significant. The strains measured were reasonably linear with load. Since the gauge length did not encompass a splice, the readings provided an indication of the axial stress in the board. Taking $E = 1.5 \times 10^6$, a stress of 500 psi at the maximum load is indicated. This is roughly double the average stress across the section.

Since the board on which the gauge was located was one of two that ran between the windows without interruption, this value of stress clearly shows that the forces were concentrating along that path.

The results of the SR-4 gauges were invalid. Whether this was due to the peculiarities of stress in the strap or the malfunctioning of the gauges was not determinable. The latter is suspected to be the case.

General Observations

The strength of a diagonally sheathed timber diaphragm is merely the integrate strength of the nails. Nowhere is the lumber loaded to more than a fraction of its ultimate strength. The rigidity of the diaphragm likewise derives from the nails.

The overall horizontal deflection of the roof with respect to the end walls was just over 3 inches as measured from the residual position of the building just prior to the last load cycle. The total deflection at the center measured from the initial zero was 5 inches. This represents 1/480th of the span, well within the allowable limit for a plastered wall. Also, such a deflection in proportion to the wall height would lie within reasonable limits for the support of a masonry wall cantilevered from a wall footing.

Since in any redundant structure the loads tend to be carried along the more rigid paths, the boards without splices or with fewer splices will tend to carry the diagonal forces. They will do so up to their ultimate capacity. This points out a need for engineers to give some consideration to the splice arrangement. In the back wall the boards that were continuous from sill to plate carried more than their proportionate share. This was true in spite of the provision in the design for carrying the loads around the windows by stiffened boundary members. A more conventional design might well have shown an even greater tendency for the forces to concentrate.

It was not possible to account for the total building deflection in terms of the local strains. The analysis did show that the greater deflection occurred in the web rather than the chords. The two ways that were employed to calculate the bending deflection did not show completely consistent results but did serve to establish an upper limit on this form of deflection.

The behavior of the structure in many respects reflected the concepts of the designer. A different concept of design may have resulted in quite a different load deflection pattern. For example, the Wagner beam analysis resulted in a stiffer chord than might otherwise have been provided.

It was not possible to determine definitely and quantitatively various contributions to the stiffness of the diaphragm. This could well be the objective of a comprehensive research program. A better understanding of the mechanism of resistance of each element individually would lead to a better understanding of the gross structure.

The economic advantages of utilizing the diagonally sheathed diaphragm up to its full potential are such that it should be designed with the same attention to details as any other composite structure.

CONCLUSIONS

No conclusion can be made concerning the ultimate strength of the timber diaphragm. The roof diaphragm, as designed, successfully withstood an average shear of 585 lbs. per foot and the end walls 1400 lbs per foot. The ability of the structure to withstand such high shears without any evidence of

failure was partly due to the attention paid in the design to the individual elements, and in particular, to the boundary members at the edge and around windows and door openings. The details employed were not representative of those commonly used. Therefore it would be unwise to conclude that these values could be obtained for any diagonally sheathed diaphragm of comparable proportions. The distribution of forces throughout a timber diaphragm will depend on the proportioning of the individual elements and connections. The loads will tend to travel through the more rigid paths as in any highly redundant structure.

The analysis of the test data led to several specific conclusions concerning the deflection characteristics of diagonally sheathed timber diaphragms..

1) The elasticity of the wood is inconsequential.
2) The bending deflection occurs as a result of splices in the chord. If the chord were continuous as provided by a masonry or concrete wall, bending deflection would be negligible.
3) The largest component of deflection will arise from shear in the web regardless of the length-depth ratio of the diaphragm.
4) The shear deflection will be due to two sources:
   a) Slip in the splices and boundary nailing of the sheathing boards in the direction of the sheathing. This component of deflection will be independent of the width of the building, but directly related to the number of boards per sheathing line and to the intensity of load along that line.
   b) Deformation normal to the direction of sheathing. This may make the larger contribution to the building flexibility. It will depend on the movement of the joist connections, the transverse slip of the intermediate nails, the bending stiffness of the boards and joists, the bending stiffness of the chord members, and finally the rotational stiffness of the intermediate nail couples.
5) The stiffness of the roof diaphragm on the building tested was greater than needed. The side walls were stucco, which would limit the allowable deflection to less than 1/360 of the span. The diaphragm was also sufficiently rigid to support a cantilevered masonry or concrete wall.

NOMENCLATURE

A - Cross sectional area
a - Panel length
b - Width of building
E - Modulus of elasticity
G - Modulus of rigidity
I - Moment of inertia
K_1 - Elastic constant corresponding to EI in the flexure formula
K_2 - An elastic constant
K_3 - An elastic constant
L - Length of building
M - Bending moment
P - Panel point load
V - Total shear in a section
v - Unit shear
δ - Local deflection or deformation
Δ - Horizontal deflection of the diaphragm

ACKNOWLEDGMENTS

The authors wish to acknowledge the help of all those who participated in the test. There were so many instruments to read that everyone present at the test had some share in the work. The contractor owner of the building, H. Coleman, was most cooperative in permitting the test to be conducted. The West Coast Lumberman's Association contributed funds to cover incidental expenses. Hathorn and Associates did the timber framing. Warth Bros., steel fabricators, loaned the tie rods. The Arch Rib Truss Company supplied the anchorages and loaned a few of the dial gauges. The California Institute of Technology supplied some dial gauges. The University of California, Los Angeles, provided the jacks, transits, and remaining dial gauges and the SR-4 gauges.

The authors wish to extend their thanks to these organizations, as well as to the many individuals too numerous to list, who contributed in any way to its success.